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DAM MODIFICATIONS CHECKED BY HYDRAULIC MODELS

By E. S. Harrison and Carl E. Kindsvater,
Members, ASCE

HYDRAULICS DIVISION

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

DAM MODIFICATIONS CHECKED BY HYDRAULIC MODELS

By E. S. Harrison[1] and Carl E. Kindsvater,[2]
Members, ASCE

Synopsis

Reconsideration of the maximum design flood, and a review of the stability analyses for the 25-year-old Bartlett's Ferry Dam, on the Chattahoochee River near Columbus, Ga., showed that the dam was deficient in freeboard and stability. Furthermore, a review of assumptions made for the original design indicated that, with the reservoir at normal pool level or above, the concrete spillway section was probably subject to tensile stresses. Field measurements also showed that high uplift pressures existed within the dam. Measures undertaken to remedy these conditions are described herein. Structural modifications, which were completed in 1950, included the installation of drains, the raising and strengthening of the non-overflow sections of the dam, and the addition of prestressed, reinforced concrete buttresses to the spillway section. Questions regarding the effect of the latter modification on the hydraulics of spillway operation called for extensive hydraulic model studies. Models were used also to obtain information pertinent to the structural design of the buttresses and certain appurtenances. Results of the uplift measurements, a description of the structural modifications to the dam, and a discussion of the model tests are given in this report.

Design Flood and Stability Investigations

The investigation reported in this paper was started in 1946, when a power company in Georgia initiated studies to determine the adequacy of spillway capacity and the structural stability of eight of its major dams. These dams

Note.—Written comments are invited for publication; the last discussion should be submitted by October 1, 1953.

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were designed and built between 1900 and 1926, at which time rainfall and flood-flow data were meager. Longer hydrologic records show, for most sections of the United States, rainfall and flood flows much larger than those shown by records available at the time the dams were built. Later methods of analysis applied to the long-term records resulted in greater design floods than those used in the original designs. Thus, in reviewing the hydrology of these projects, the power company sought to determine the adequacy of the dams in the light of the greater storms that have occurred since their construction. Only those storms experienced east of the Mississippi River were considered.

Of the eight dams, only Bartlett's Ferry Dam was shown to require remedial measures. Here, a revised design flood resulted in a calculated reservoir level which would reach the top of the earth non-overflow sections. Stability

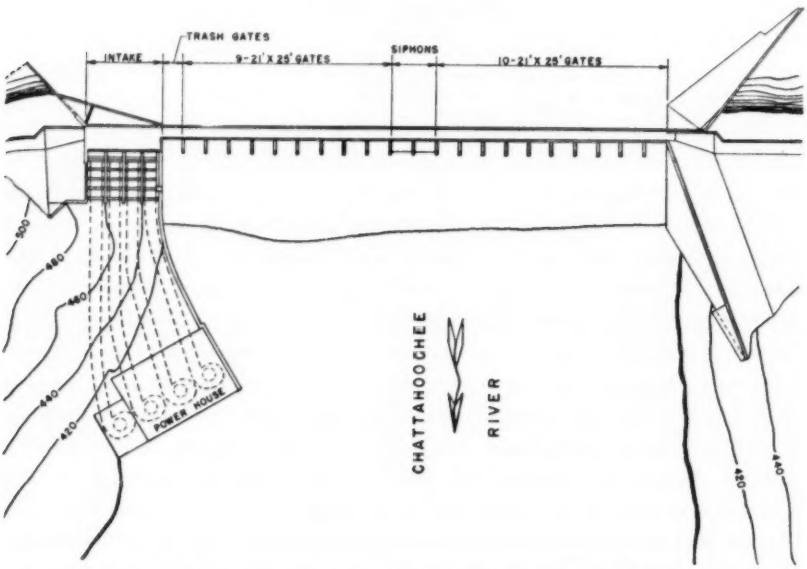


FIG. 1.—GENERAL PLAN OF THE ORIGINAL STRUCTURE, BARTLETT'S FERRY DAM NEAR COLUMBUS, GA.

computations based on modern practice also indicated that the spillway section should be strengthened to provide adequate safety. Furthermore, observations in the field indicated that excessive uplift pressures existed in this section of the dam. It was apparent that immediate remedial measures were needed.

DESCRIPTION OF BARTLETT'S FERRY DAM

Bartlett's Ferry Dam was built in 1925, and was purchased by a power company in 1929. It consists of a concrete gravity spillway section, a concrete intake section, and two earthfill abutments. The maximum height of the spillway section, measured from the river bed to the normal pool level, is

approximately 140 ft. A plan of the development is shown in Fig. 1. This dam is described in greater detail elsewhere.[3,4,5,6]

Fig. 2 shows a cross section of the spillway of the dam as originally built. This figure also illustrates the uplift assumption used in the original design. It was assumed that the uplift would vary from two thirds of full headwater at the upstream face of the dam to no uplift at the longitudinal drain, which was 15 ft downstream from the reservoir. No allowance was made for the uplift within the body of the dam. Below El. 400, as illustrated in Fig. 2, base widths used in the design were increased by including a part of the spillway bucket. These increased widths were based on an assumed change in slope beginning at El. 400. Because of the latter assumption in the original design,

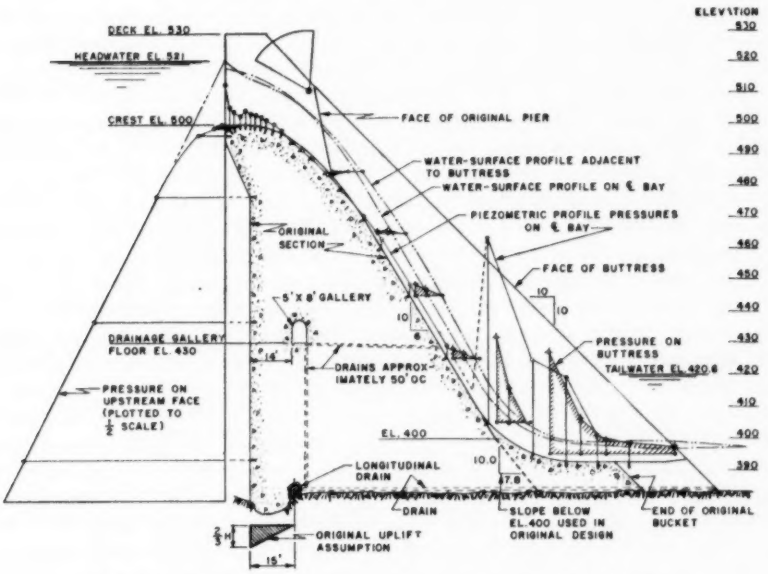


FIG. 2.—SECTION OF THE SPILLWAY, SHOWING MODIFICATIONS TO THE ORIGINAL STRUCTURE AND PIEZOMETRIC PROFILES FOR THE MODIFIED SECTION

and because no allowance was made for uplift above the foundation line, review computations which included uplift indicated that the critical plane from the standpoint of stability was at El. 400. Here, assuming a 50% uplift, the calculated stress at the upstream face was approximately 27 lb per sq in. tension. This stress value is based on the assumption that the reservoir would be at normal level, El. 521, which corresponds to the elevation of the top of the spillway gates.

[3] "Water Power Engineering," by H. K. Barrows, McGraw-Hill Book Co., Inc., New York, N. Y., 1943, p. 524

[4] *Ibid.*, p. 536

[5] *Ibid.*, pp. 744–748.

[6] "The Bartlett's Ferry Hydroelectric Development," by H. A. Hageman and T. B. Parker, *Journal*, Boston Soc. of Civ. Engrs., March, 1926, pp. 93–125

Because there was every reason to believe that uplift water pressures existed in the concrete dam above the rock line, immediate steps were taken to relieve this pressure as much as possible. Diamond drill holes of 3-in. diameter were drilled from the drainage gallery, at El. 430, into the rock foundation. The purpose of these drill holes was to relieve the existing internal water pressure, and, also, to determine, if possible, what the existing water pressures were. The internal water pressure conditions revealed by these holes were unusual.

Uplift Pressures in the Spillway Section

Thirty-one 3-in. drain holes were drilled vertically downward from the gallery in the spillway section of the dam. In fifteen of the holes, the drill struck water in such large quantities and pressures that it was necessary to discontinue drilling in these holes before they were completed. This necessitated dividing the work into two stages. The depths drilled in Stage 1 are indicated by solid lines in Fig. 3. This diagram also shows the locations, the depths drilled, the results of the tests, and the observations for Stage 1. Water flowed out of the top of twenty-six of the thirty-one holes drilled. Upward flows varied from seepage to a solid 3-in. jet rising 59 in. above the top of hole No. 52. The flow from all holes was about 3,400 gal per min. The water pressure was also measured at the holes. The measured pressures, expressed in percentage of full headwater pressure, are shown in the tabulation of Fig. 3.

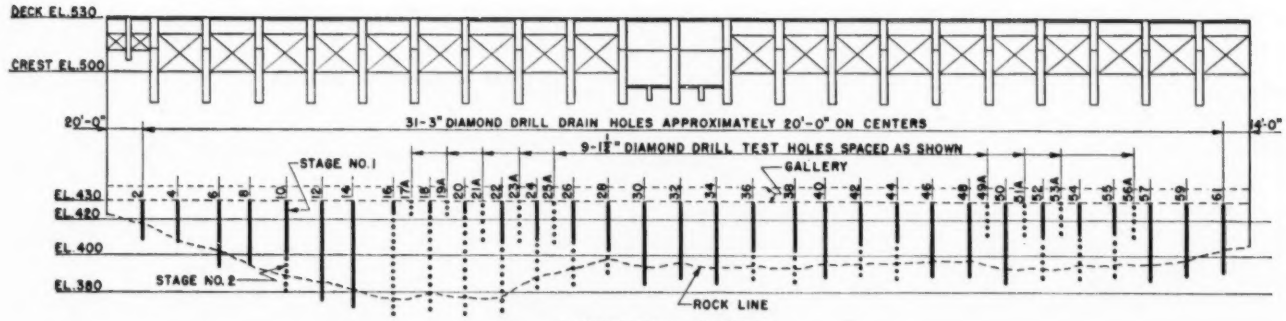
The work of Stage 2 consisted of pressure-grouting the holes that had excessive discharges, re-drilling these holes, and extending them into the rock foundation. Fig. 3 shows the final depths, the bags of cement used in grouting, and the water pressures in terms of full headwater after the holes were grouted and re-drilled. The tabulations show a comparison of the flows and pressures measured for the conditions of Stage 1 and Stage 2. This tabulation shows that, after the drilling of Stage 1, seventeen holes had spouting flows, eleven holes had pressures equal to full headwater, and twenty-one holes had pressures exceeding $\frac{1}{2}$ full headwater pressure. After the grouting and re-drilling (Stage 2) was completed, none of the holes had excessive flows. Only one of the holes had a pressure which exceeded $\frac{1}{2}$ full headwater, and only five had pressures which exceeded $\frac{1}{3}$ full headwater. It is notable that none of the holes drilled during either stage indicated excessive leakage at the rock line.

Fig. 3 shows nine holes designated by numbers and by the letter A. These were diamond drill holes 1.5 in. in diameter, drilled midway between those 3-in. holes which were grouted. The purpose of these holes was to test the effectiveness of the grouting. In every case, only small flows were encountered in the test holes. In one, a grout core approximately $\frac{3}{16}$ in. thick was recovered.

Holes of 3-in. diameter were also drilled from the crest of the dam to the drainage gallery. None of these holes indicated excessive leakage.

Strengthening of the Dam

The modified dam is designed to withstand a reservoir level 8 ft higher than that used in the original design. At this higher pool level the estimated maximum spillway discharge is 337,000 cu ft per sec, which is 68% larger



LONGITUDINAL SECTION

HOLE NUMBER	STAGE NO. 1				STAGE NO. 2				
	DEPTH DRILLED	HEIGHT JET (1)	DEPTH TO WATER (2)	MEASURED PRESSURE (3)	BAGS OF CEMENT	DEPTH DRILLED	HEIGHT JET (1)	DEPTH TO WATER (2)	MEASURED PRESSURE (3)
2	21'-8"	S(4)	...	69	...	...	S	...	35
4	21'-8"	S	...	0	...	...	S	...	0
6	37'-0"	S	...	59	...	...	S	...	2
8	35'-4"	...	27'-5"	...	...	...	...	27'-0"	...
10	29'-0"	11"	...	100	28	49'-6"	S	...	9
12	54'-6"	1/2"	...	61	...	...	S	...	0
14	59'-0"	...	27'-5"	...	...	...	...	27'-1"	...
16	7'-0"	11"	...	100	50	64'-0"	...	18'-6"	...
17A	...	...	...	...	...	8'-0"	S	...	...
18	6'-8"	25"	...	100	75	60'-6"	...	27'-1"	...
19A	...	...	...	...	...	8'-0"	1/4"	...	...
20	6'-8"	17"	...	100	20	63'-0"	S	...	0
21A	...	...	...	...	17	22'-0"	1/4"	...	...
22	21'-8"	8"	...	80	55	61'-5"	S	...	49
23A	...	...	...	...	...	23'-0"	2 1/2"	...	...
24	21'-8"	57"	...	100	67	49'-0"	S	...	14
25A	...	...	...	...	...	23'-0"	S	...	...
26	21'-8"	7"	...	100	30	46'-8"	S	...	2
28	26'-8"	2 1/2"	...	100	15	40'-0"	...	27'-0"	...
30	46'-2"	...	24'-2"	...	...	...	...	19'-8"	...
32	43'-0"	...	8'-7"	...	...	...	...	2'-0"(6)	...
34	46'-0"	...	8'-7"	...	...	...	...	6'-0"	...
36	26'-8"	4 1/2"	...	77	15	43'-0"	S	...	0(7)
38	26'-8"	9"	...	91	25	45'-3"	1/4"	...	49(8)

HOLE NUMBER	STAGE NO. 1				STAGE NO. 2				
	DEPTH DRILLED	HEIGHT JET (1)	DEPTH TO WATER (2)	MEASURED PRESSURE (3)	BAGS OF CEMENT	DEPTH DRILLED	HEIGHT JET (1)	DEPTH TO WATER (2)	MEASURED PRESSURE (3)
40	43'-0"	13/4"	...	93	...	...	S	...	9(9)
42	21'-4"	3"	...	53	7	41'-4"	...	27'-0"	...
44	21'-8"	2"	...	100	20	42'-6"	S	...	0(5)
46	41'-0"	4"	...	91	...	...	S	...	46
48	41'-0"	1"	...	29	...	...	...	27'-6"	...
49A	...	...	...	...	...	19'-0"	S	...	0
50	45'-4"	31/2"	...	91	...	...	S	...	69
51A	...	...	...	...	...	18'-6"	S	...	0
52	17'-7"	59"	...	100	50	42'-8"	S	...	0
53A	...	...	...	...	...	18'-6"	...	3"	...
54	17'-0"	47"	...	100	35	43'-3"	S	...	0
55	17'-0"	42"	...	100	50	40'-6"	S	...	0
56A	...	...	...	...	...	18'-6"	...	12'-9"	...
57	42'-0"	S	...	0	...	...	S	...	0
59	39'-10"	S	...	0	...	...	S	...	0
61	36'-8"	S	...	0	...	...	S	...	0

NOTES: (1) ABOVE TOP OF HOLE. (2) BELOW GALLERY FLOOR. (3) PERCENT OF FULL HEAD. (4) S - SEEP. (5) PRESSURE 29% WITH 46 CLOSED. (6) SMALL FLOW WITH 34, 36, 38 & 40 CLOSED (PRESSURE 0%) (7) 32% WITH 38 & 40 CLOSED. (8) 60% WITH 36 & 40 CLOSED. (9) 35% WITH 34, 36 & 38 CLOSED.

FIG. 3.—PLAN AND LOG OF DRILLING OPERATIONS

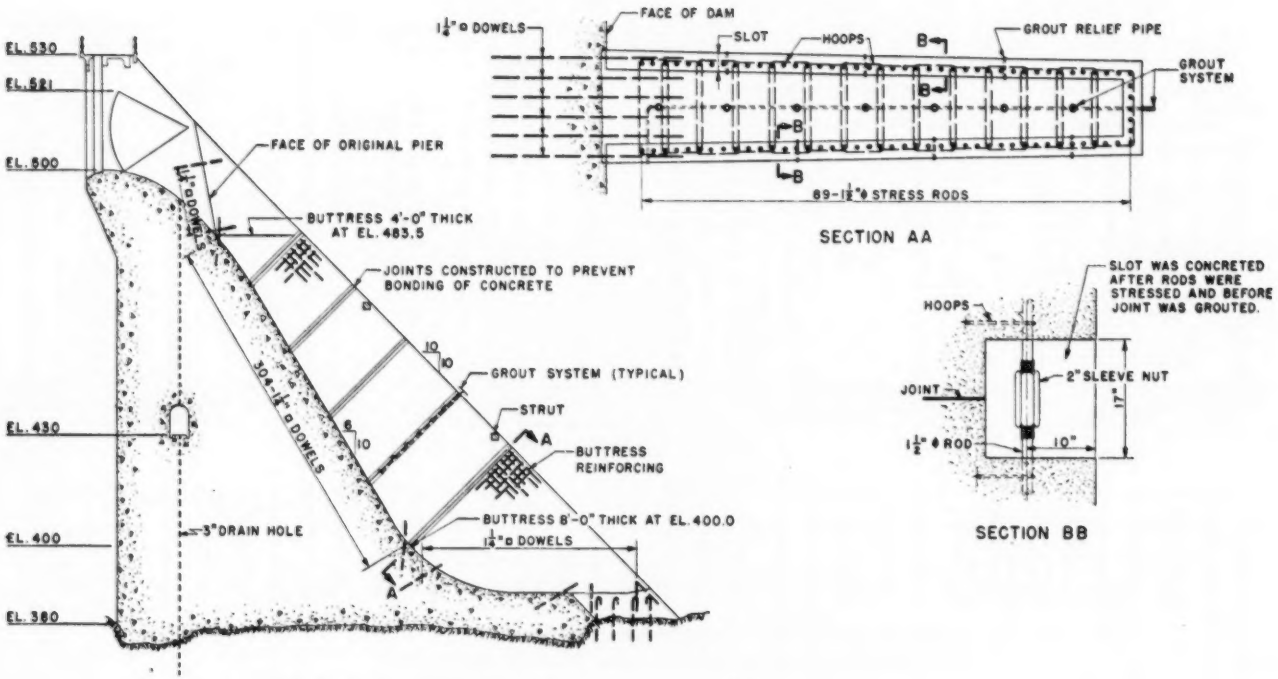


FIG. 4.—DETAILS OF SPILLWAY BUTTRESS CONSTRUCTION

then the original design discharge. To meet these new requirements, the earth abutments were raised 5 ft and were strengthened by adding rock fills. Wave walls 4 ft high were added to the tops of the earth sections.

Strengthening of the spillway section of the dam was required because of the possible large tensile stresses revealed by the uplift measurements and the stability analyses. The structural modification adopted to accomplish this purpose consisted of adding prestressed, reinforced concrete buttresses to the downstream side of the spillway, one at each of the existing crest-gate piers. Fig. 4 shows certain details of the buttresses, and Fig. 5 shows the spillway after modification.

The design of the modified spillway section was based on the following considerations: (1) The buttresses would be added to the dam while it was in a stressed condition; (2) after completion, the buttresses would be made to carry a predetermined part of the load by means of adjustable stress rods embedded

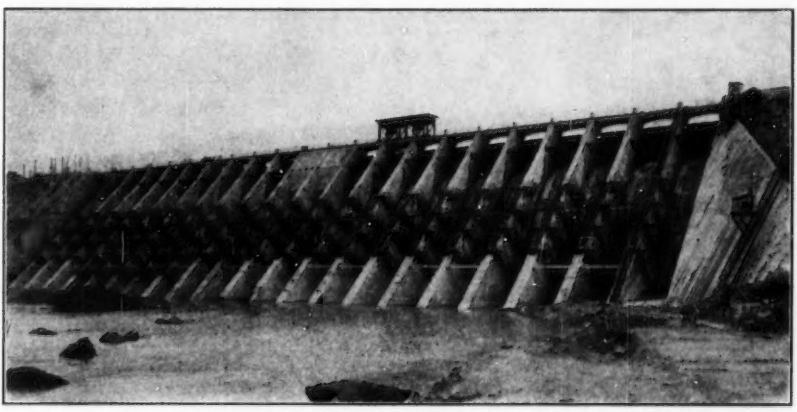


FIG. 5.—THE MODIFIED STRUCTURE BEFORE REMOVAL OF THE ACCESS STAIRWAYS

in the buttresses; (3) the total shear stress between the existing dam and the buttresses would be carried by steel dowels; and (4) the uplift pressure, at any horizontal plane, would be assumed to vary from 50% of full headwater to zero pressure, or tailwater, and would be assumed to be applied to 100% of the base area.

The design requirement that the buttresses be prestressed was imposed by the apparent large tensile stresses in the existing dam when subjected to normal reservoir levels, and by the fact that the reservoir could not be lowered enough to reduce these existing stresses to values permitting the construction of unstressed buttresses that would eliminate tensile stresses with maximum headwater conditions.

The principal steps used in the stability analysis of the modified structure involved the determination of (1) stresses in the existing structure with lowered headwater, (2) stresses induced by the addition of the buttresses, (3) stresses induced in the modified structure by loading of the buttresses, (4) stresses in-

duced in the modified structure by raising the headwater to maximum level, and (5) the combination of all conditions to determine the maximum stresses.

The conventional elastic theory was used in the design. Allowable unit stresses used for the design of the buttresses were purposely kept low (one half the usual values) to provide large factors of safety.

Fig. 4 shows the general details of the buttresses. The buttresses were added in separate blocks, as indicated. The top of each block was formed to provide a smooth surface and was oiled to prevent bonding to the next lift. Steel stress rods, with sleeve nuts at each lift joint, were installed for the purpose

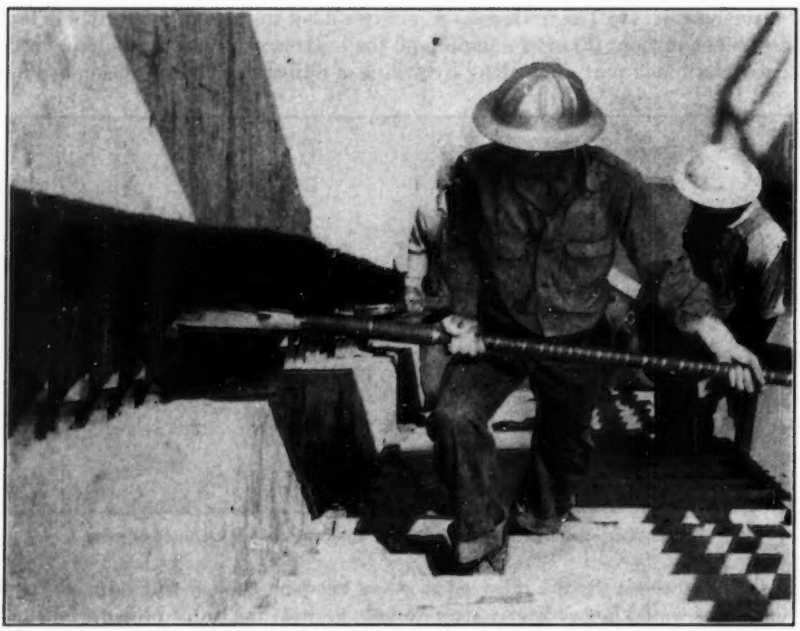


FIG. 6.—WORKMEN SHOWN TIGHTENING STRESS NUTS IN ONE OF THE BUTTRESSES DURING PRESTRESSING OPERATIONS

of prestressing the buttresses after they were completed. The buttresses were anchored to the existing dam by grouting 1.25-in. steel dowels 8 ft and 10 ft into the old gravity dam. These dowels were designed to carry the total shear at the contact line between the old dam and the buttresses, no allowance being made for the development of shear between the old and the new concrete. The old concrete was chipped down an average of 6 in. under areas of new construction. The temporary stairways shown on the sides of the buttresses in Fig. 5 were provided to give access to the stress nuts. Fig. 6 shows workmen on one of these stairways during the stressing operations.

Prestressing the Spillway Buttresses

During the period when the buttresses were under construction, the reservoir was drawn 11 ft below normal pool level. After their completion, the reservoir was lowered an additional 10 ft, and the buttresses were then loaded to predetermined values, because it was not feasible to lower the reservoir further, the existing dam remained in a stressed condition, and, for this reason, it was necessary to prestress the buttresses so that they might assume their assigned part of the total load.

Stress was applied to the buttresses by means of the stress (sleeve) nuts. This operation involved a systematic tightening of the nuts with torque wrenches previously calibrated by a testing laboratory. Ten sets of rods and sleeve nuts, duplicates of those in the buttresses, were used in laboratory calibration in order to determine the uniformity of thread friction. The tests showed a very consistent relationship between the rod load and the wrench-dial readings. To insure an even distribution of stress in the rods in all the buttresses, the torque-wrench loads were applied in increments. An initial loading of 50% of the predetermined final stress was applied to all rods. In successive steps, loads equivalent to 75%, 85%, 90%, and progressively higher percentages of the full load were applied. A final check was made at each sleeve nut before the access slots were filled with concrete. The last operation consisted of pressure-grouting the buttress lift joints, and the joints between the buttresses and the face of the original structure. An embedded grout-pipe system, shown in Fig. 4, had been provided for this purpose. A considerable amount of grout was injected into all the lift joints. Very little grout was taken by the face joints.

Necessity for Model Tests

When the planning studies for the modification of the spillway section of the dam indicated that the buttress plan might be the most economical, some uncertainty arose concerning the effects of the buttresses on flow conditions in the area below the spillway. Although a negligible amount of erosion had occurred during the life of the original structure, it was suggested that the "nozzling" effect of the buttresses might cause excessive scour below the dam. It is to be noted that the effective flow width at the bottom of the spillway was reduced approximately 28% by the addition of the buttresses. Continued studies raised other questions, including the magnitude of the hydraulic forces on the buttresses, the design of the spillway bucket extensions required to accommodate the buttresses, the height and location of construction cofferdams, and the coefficient of discharge for the modified spillway. The need for model studies was clearly indicated, and arrangements were made to conduct these studies in the hydraulics laboratory at the Georgia Institute of Technology, at Atlanta.

Background for Model Studies—Prototype Experiences

A complete understanding of the model test program and results is not possible without a brief review of certain unusual prototype operating condi-

tions and experiences. As Bartlett's Ferry Dam had been in operation for more than twenty years, it was possible to interpret the model observations in the light of those recorded experiences, and the resulting conclusions were especially significant.

As constructed, the original structure did not include the customary elaborate provisions for the development of a stilling basin below the spillway, despite the abnormally low tailwater depths which occur at the site. Thus, for almost all conditions of operation, the tailwater was swept off the spillway bucket, exposing foundation rock near the toe of the dam to high-velocity flows. It is remarkable, however, that during the twenty-five years of the dam's existence, no significant movement of the foundation rock had taken place, and erosion had never been a serious problem.

During the time that the project had been in operation, several large floods were experienced. These included the flood of March, 1929, for which the peak spillway discharge was estimated to be approximately 135,000 cu ft per sec. Of greater significance, perhaps, is the fact that several gates had been opened during a considerable part of each year in order to pass normal flows. Early observations during high flows at the dam indicated that excessive spray and wave action made it undesirable to open spillway gates in the vicinity of the powerhouse. Following the flood of 1929, the powerhouse training wall was raised 5 ft to prevent overtopping during high flows.

The normal power-pool level for the Bartlett's Ferry Project is at El. 521, which corresponds to the top of the spillway crest gates. The revise maximum flood-pool level is at El. 530. Maximum spillway discharge capacities are computed on the basis of 19 full spillway bays, 2 trash gates, and 4 siphons, all in operation. The combined capacity of the trash gates and siphons is very small. The siphons, which occupy a width equal to 2 spillway bays, are normally closed, but can be opened for flood discharge. In recent years, the last gate on the east end of the spillway has been opened infrequently, in order to avoid concentrations of the flow along the foot of the retaining wall.

The 1/80-Scale Model

Two models were constructed, the first of which was a 1/80-scale comprehensive model of the spillway and powerhouse sections of the dam, plus a length of the river extending approximately 700 ft (prototype) downstream from the dam. For purposes of the model tests, the spillway crest gates were numbered from 1 to 19, beginning at the powerhouse end of the dam, as shown in Fig. 9, appearing subsequently. One bay, adjacent to the powerhouse, which is occupied by trash gates, was reproduced as an additional spillway bay, designated by the letter T. Two bays representing the siphons were designated S_1 and S_2, reading from west to east. The models of the proposed buttresses, each of which required special fitting to conform to the irregular spillway bucket, were assembled as a single unit to facilitate installation. River bed topography was reproduced in concrete except for scour tests, during which a short section of the bed immediately below the spillway was composed of pea gravel. Fig. 7, illustrating the original and modified structure, shows the completed 1/80-scale model.

The objectives of tests on this model were (1) to contrast flow conditions below the original spillway structure with conditions which might occur if the buttresses were added; (2) to develop a satisfactory design for the spillway bucket extension; (3) to determine a satisfactory gate-opening sequence for passing floodwaters over the spillway; (4) to explore the necessity for raising the original powerhouse training wall, and to develop a satisfactory design for this modification, if required; and (5) to determine the best location and height of cofferdam for the two-stage construction of the buttresses.

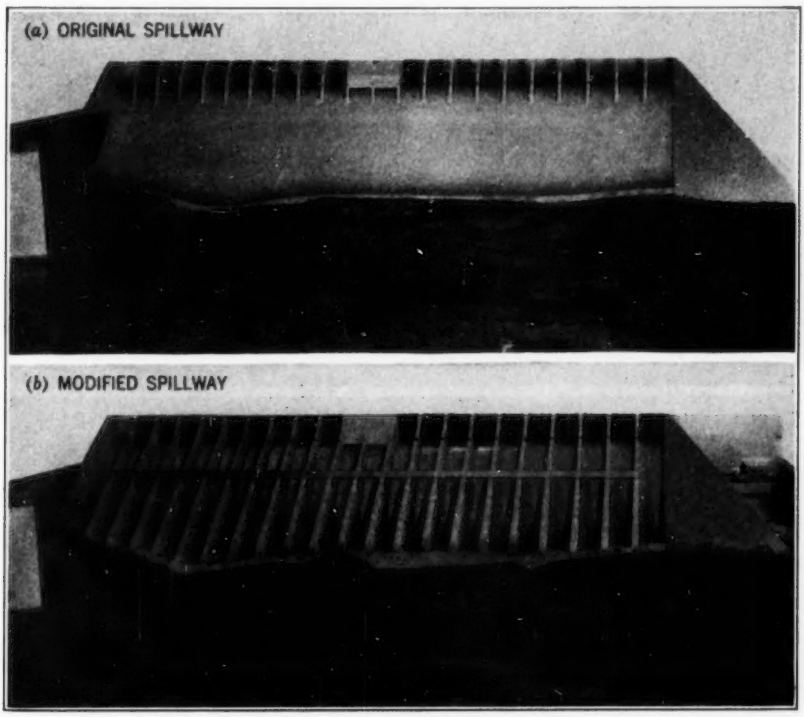


FIG. 7.—MODEL BUILT TO SCALE OF 1/80

In this limited space, only the more important test results, with little of the supporting data, can be examined.

Effect of the Buttresses on the Stilling Basin.—Tests on both the movable-bed model and the fixed-bed model indicated that, in general, erosive tendencies would be lessened by the addition of buttresses to the original spillway. The fairly obvious explanation of this conclusion lies in the observed effect of the buttresses in dividing the nappe into separate jets, which discharge freely off the spillway bucket, falling on the tailwater blanket a considerable distance downstream from the end of the dam. In contrast, tests on the original

bucket indicated that the lack of free air below the continuous sheet of water passing across the bucket resulted in negative pressures under the nappe, unstable flow, and a lowered trajectory. When the results of the model tests were compared with the recorded performance of the original structure, it could be concluded that conditions in the stilling basin would not be unfavor-

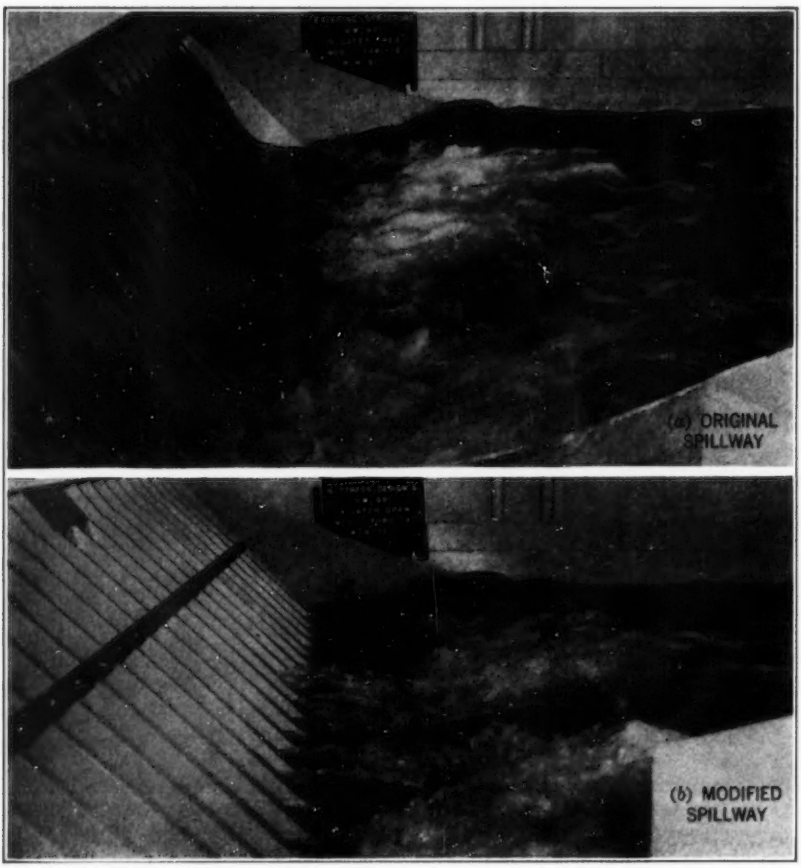


FIG. 8.—A DISCHARGE CORRESPONDING TO 172,000 CU FT PER SEC

ably affected by the proposed construction. Fig. 8 illustrates the characteristic differences in behavior of the original and the modified spillways. Typical results of comparative scour tests are shown in Figs. 9 and 10. In all the tests illustrated by Figs. 8, 9, and 10, the model was operating at normal reservoir level, at El. 521, with all gates opened wide, corresponding to a prototype flow of 172, 000 cu ft per sec.

Design of Spillway Bucket Extension.—As shown in Fig. 2, the spillway bucket on the original structure was of a minimum length and did not include any type of end sill or baffle blocks. As shown in Fig. 4, the bucket had to be lengthened from 13 ft to 23 ft (variable because of different apron elevations across the length of the spillway) to accommodate the added buttresses. Thus, an important objective of the model tests was to develop a satisfactory design for the apron extension. The only type of sill considered for comparison with a horizontal extension was a simple "flip" sill with plane faces sloping up in the downstream direction. The design that was adopted finally called for the construction of a sill 1 ft high and 6 ft in base width in every gate bay except Nos. T, 1, 2, and 3 at the powerhouse end of the spillway. This design appeared to give optimum performance, considering scour, wave action, and spray. The sills were omitted in the 4 bays nearest the powerhouse in order not to increase the objectionable spray which normally occurs as a result of the unfortunate orientation of the powerhouse with respect to the spillway. Sills in the bays nearer the opposite end of the spillway, where the bucket is much higher, caused considerable spray in the model, but greatly reduced scour around the retaining wall. As these sills are far removed from the powerhouse, spray was not believed to be a serious consideration. Fig. 10 shows the results of a typical scour test on Design C, which differs from the final design only in the inclusion of sills in bays T to 3, inclusive.

Gate-Opening Sequence.—The problem of determining a most satisfactory gate-opening sequence for floodwater release was, of course, limited to flows less than the full capacity of the spillway at top-of-gates level, El. 521. The criteria for evaluating various gate-opening arrangements included surface currents, waves, spray, and stilling-basin scour. The schedule of operation as finally developed necessitated opening the gates in the following sequence (see Fig. 9): Nos. 10, 12, 14, 16, 18, 9, 8, 7, 6, 5, 11, 13, 15, 17, 19, 4, 3, 2, 1, T. It was indicated that the siphons could be opened at any time. The adopted sequence obviously develops quite logically, but it was made clear during the tests that undesirable flow conditions frequently result from gate arrangements which, without the benefit of the model studies, would appear to be just as logical. In the model, as in the prototype, it was demonstrated that gates adjacent to the powerhouse should be opened last.

Other Objectives.—A secondary objective of tests on the 1/80-scale model, which developed as the tests proceeded, involved the raising of the powerhouse training wall to resist and to deflect spray and overriding flow from spillway bays near the powerhouse that normally would be opened only during extreme floods. The final design for this modification necessitated raising the wall 4 ft and adding a 4-ft overhanging extension at the top to serve as a wave deflector. It is important to note that the necessity for this modification was due primarily to powerhouse orientation and not to the addition of buttresses on the spillway.

The height and location of cofferdams required for two-stage construction of the spillway buttresses was also determined in brief tests on the 1/80-scale model. A unique result of this study was the enlargement of the cofferdam to permit flooding of, and discharge into, the cofferdam areas in the event of

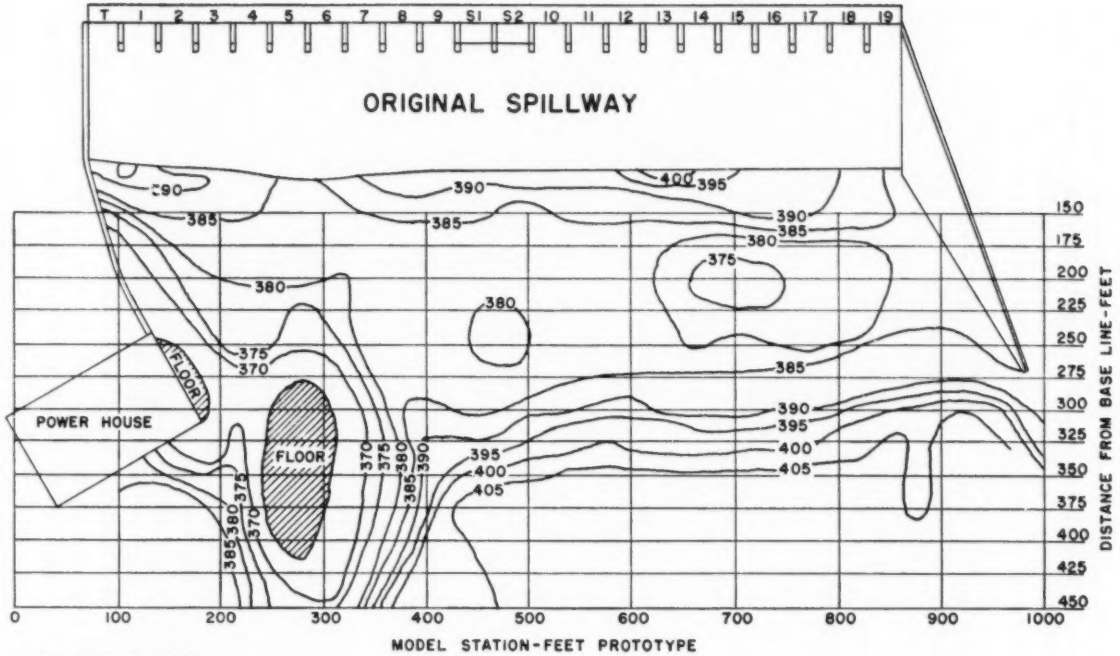
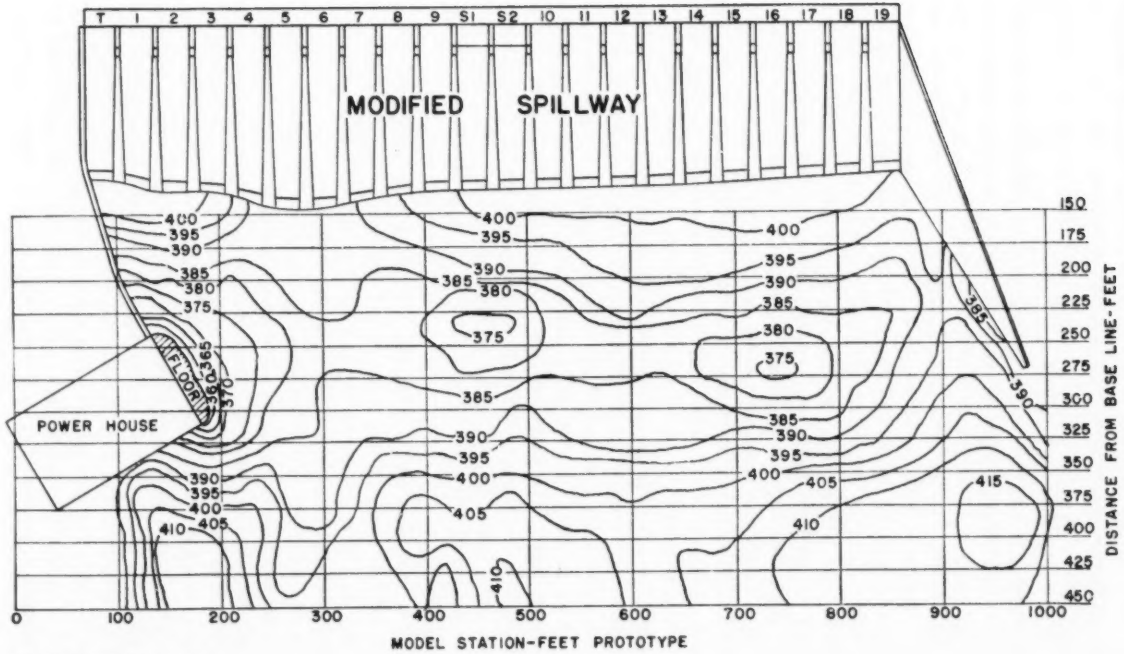


FIG. 9.—CONTOUR MAP OF ERODED BED FOR THE CONDITION SHOWN IN FIG. 8(*a*)



HEADWATER EL. 521.0
TAILWATER EL. 420.6
DISCHARGE 172,000 CFS
GATES 1–19 PLUS T, S1 & S2 OPEN
DESIGN C WITH 1'X 6' END SILLS

FIG. 10.—CONTOUR MAP OF ERODED BED FOR THE CONDITION SHOWN IN FIG. 8(b)

the occurrence of a flood larger than could be passed through the unenclosed spillway bays. Fortunately, construction operations at the dam site did not have to be interrupted for a prototype check of this feature.

The 1/40-Scale Model

The second model was a 1/40-scale sectional model of one full bay and two adjacent half-bays of the spillway. The crest section of this model was constructed accurately in Lucite, and fifty piezometers were located on the spillway and buttresses for pressure measurements. The model was not equipped with a movable bed for scour tests. The principal objectives of tests on this model were (1) the measurement of hydraulic pressures on the dam and buttresses; (2) the recording of water-surface profiles for various conditions of flow over the modified spillway; and (3) the calibration of the modified spillway for free discharge and various gate-opening arrangements.

Pressures on Dam and Buttresses.—Fig. 2 includes the results of a typical test to determine pressures on the dam and buttresses. As indicated, piezometers were located on the center line of one bay of the spillway, and on the face of one buttress. Pressure data are plotted as piezometric profiles, with straight lines drawn between plotted peizometric levels. It is indicated that positive pressures exist over the entire spillway except for a short reach below the crest. Considerably higher negative pressures were recorded with the headwater at maximum flood level, at El. 530. In the latter case, a maximum negative pressure head of (−) 6.40 ft was recorded at the highest point on the spillway. Of course, very high pressures were observed in the bucket and in the adjacent faces of the buttresses. Stresses in the buttresses due to hydraulic forces, however, were insignificant in the structural design. Tailwater had no effect on the measured pressures because it was washed off the bucket for all conditions of operation.

Surface Profiles and Discharge Calibration.—Typical water-surface profiles along the center line of one bay and adjacent to the face of one buttress are also shown in Fig. 2. This information was used, in part, to insure adequate clearance for the lateral struts located between the buttresses.

The spillway was calibrated for the following conditions of gate operation: (1) All gates were open; (2) the center gate was open and both adjacent half-gates were closed; and (3) the center gate and one half-gate were open, and one adjacent half-gate was closed. As the gates are never used for part-gate operation, all tests were made for free-discharge conditions. The coefficients of discharge (C) computed on the basis of the simple weir formula had an average value of 3.7 at normal pool level and approximately 4.2 at maximum flood level. The weir formula may be expressed as $Q = C\ L\ H^{\frac{1}{2}}$, in which Q is the discharge in cubic feet per second, L is the gross length of the openings in feet, and H is the static head in feet. The unusually high coefficient of discharge is obviously a consequence of the aforementioned negative pressure on the crest.

Acknowledgments

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the direct supervision of E. S. Harrison, M. ASCE, and under the general direction of W. P. Hammond, vice-president of the Georgia Power Company. Consulting engineers were S. H. Woodard, O. G. Thurlow, and N. F. Williams, Members, ASCE. R. R. Randolph, of Southern Services, Incorporated, assisted by engineering personnel of the Georgia Power Company, conducted the model tests under the direction of one of the writers. The laboratory studies were made in the hydraulic laboratory of the Georgia Institute of Technology by arrangement with the Georgia Tech Research Institute.

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